EARTHQUAKE ENGINEERING AND STRUCTURAL DYNAMICS, VOL. 26, 637–655 (1997)

COMPARATIVE STUDY ON THE SEISMIC RETROFIT OF A MID-RISE STEEL BUILDING: STEEL BRACING VS ENERGY DISSIPATION

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SUMMARY

The paper presents a comparative study of an existing retrofit for a mid-rise steel building using additional stiff steel braced-frames against an alternate retrofit using ADAS (Added Damping and Stiffness) passive energy dissipation devices. The subject building, located near Alameda Park in downtown Mexico City, is a ten-storey office building that was built in the 1950s. The structure was damaged during the 1985 Michoacán Earthquake because of resonant response with the site. The building was later retrofitted using additional braced frames according to the seismic provisions of Mexico's 1987 Federal District Code. The retrofit scheme was planned to take the structure away from resonant responses and to inhibit structural damage.

A proposed upgrade using ADAS energy dissipation devices was studied to compare energy dissipation against traditional stiffening using steel braces as retrofit options for mid-rise buildings in Mexico City's lake-bed zone. Different sets of analyses were carried out to compare both alternatives: (a) three-dimensional elastic analyses; (b) limit analyses and; (c) nonlinear dynamic analyses for postulated site ground motions for a $M_s = 8.1$ earthquake. Initial costs of the retrofit schemes were also studied. The comparative studies suggest that a retrofit using ADAS devices would have a better dynamic performance than the one using steel braces. However, the steel bracing retrofit provides more strength and its initial cost of retrofit is less than that of the ADAS retrofit. © 1997 by John Wiley & Sons, Ltd. Earthquake eng. struct. dyn. 26: 637–655, 1997.

(No. of Figures: 14. No. of Tables: 3. No. of Refs: 22.)

KEY WORDS: ADAS devices; passive energy dissipation; seismic retrofit; braced steel frames; ordinary moment resisting frames; non-linear dynamic analysis; initial cost of retrofit

INTRODUCTION

Crowded cities have been severely struck by damaging earthquakes during the last decade. Mexico City was severely shaken during 19, September 1985, $M_s = 8{\cdot}1$ Michoacán Earthquake. More than 4500 people died and about 16 000 were injured.[1] A total of 12 700 structures were affected, where 1778 were severely damaged or collapsed and 4826 experienced moderate damage.[1] Medium-rise, moment-resisting frame buildings were among the most severely affected because the local soil conditions of the lake-bed region of Mexico City, and the structural dynamics of these buildings led these structures to resonant responses with the ground in many cases.

Given the large number of medium-rise buildings affected by the earthquake and the need to use these facilities again as soon as possible, several retrofit techniques were used in Mexico City for their seismic upgrading.[2] A popular solution used in Mexico City after the 1985 Michoacán earthquake was the stiffening

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Contract grant sponsor: Secretaría General de Obras del Departamento del Distrito Federal.

CCC 0098–8847/97/060637–19$17·50

Received 15 January 1996
Revised 27 August 1996

and strengthening of buildings with concentric diagonal steel bracing to take these buildings away from resonant responses. This solution has been successful in the past for the seismic retrofit of RC buildings in Mexico City that survived the 1985 Michoacán Earthquake with practically no structural damage.[3–5]

Recently, some structures have been upgraded in Mexico City using energy dissipation devices. Energy dissipation devices are attractive to use because they improve the overall behaviour of the structure by increasing its internal damping through the energy dissipated by the inelastic deformation of these special devices. Consequently, the dynamic structural response is considerably reduced, particularly in the original members of the structure. Because many of these systems have to be mounted on steel braces to be attached to the original structure, the lateral stiffness of the structure is also increased, making these systems particularly attractive for buildings that are vulnerable to having resonant response with the ground in the lake-bed region of Mexico City. The Added Damping and Stiffness (ADAS) energy dissipation device, which has been extensively tested and studied at the Universities of California at Berkeley[6] and Michigan,[7] has caught the attention of Mexican structural engineers and researchers. Up to now, there are two RC buildings upgraded with ADAS devices in Mexico City.[8] In addition, a three-building complex is currently being retrofitted with this system.[8,9] For these buildings, the structural engineer used the load–deformation curves obtained experimentally by the supplier of the ADAS devices.[8] However, numerical modelling of the ADAS have recently been proposed,[6,10,11] so these numerical formulations can be used for preliminary design purposes and perhaps trigger their use in structural design.

There is interest among Mexican practicing and research engineers in knowing when energy dissipation might be more advantageous than traditional stiffening for both design and retrofit of buildings in Mexico City. There might not be a simple answer. Nevertheless, some research efforts are being conducted in this direction in idealized structural systems,[12] looking to general response quantities. There are few studies available that have expressively been conducted of retrofitted structures. The present study deals with a case study where a proposed upgrade using ADAS energy dissipation devices was compared against traditional stiffening using steel braces as retrofit options for an existing mid-rise steel building in Mexico City's lake-bed zone. Detailed analytical studies were conducted to compare both alternatives. The types of analyses and their results are discussed in the following sections.

ORIGINAL BUILDING

The subject building, located near Alameda Park in downtown Mexico City, is a ten-storey office building that was built in the 1950s according to the provisions of Mexico's 1942 Federal District Code.[13] Under this building code, the structure was classified to be type III and was designed for a seismic base shear of $0{\cdot}025W$, where W is the total weight of the structure. A plan view of the original structure is depicted in Figure 1. The plan measures approximately 61·0 m × 18·2 m. Typical bay widths were 5·55 m in the longitudinal direction and 5·85 m or 6·50 m in the transverse direction. The total height of the structure from the ground level is 33·5 m, with typical storey heights of 3·5 m, except at the first floor, which has a height of 5·5 m. The total height was close to the 35 m limit established by the 1942 code.[13]

The original steel structure consisted of Ordinary Moment Resisting Frames (OMRF) in both orthogonal directions. Typical columns are of box cross section made either with two channels riveted to two 0·95 cm (3/8″) steel plates or with four 15·24 × 15·24 × 0·95 cm (6″ × 6″ × 3/8″) angles riveted to four 1·27 cm (1/2″) steel plates. The dimensions of these columns vary considerably with height and go from 65 × 50 cm at the first storey to 35 × 32·5 cm in the upper three storeys. The beams are approximately S15 × 50 for the first three storeys, S12 × 35 from the fourth to seventh storey and S12 × 31·8 for the eighth to tenth storey. All original connections are riveted. The original foundation system is mixed and consists of a 4·8 m deep box foundation over point-bearing piles.

The original structure was later modified by adding three storeys with elements similar to the original sections for stories eight to ten. At the time of the 1985 Michoacán earthquake, the structure consisted of 13

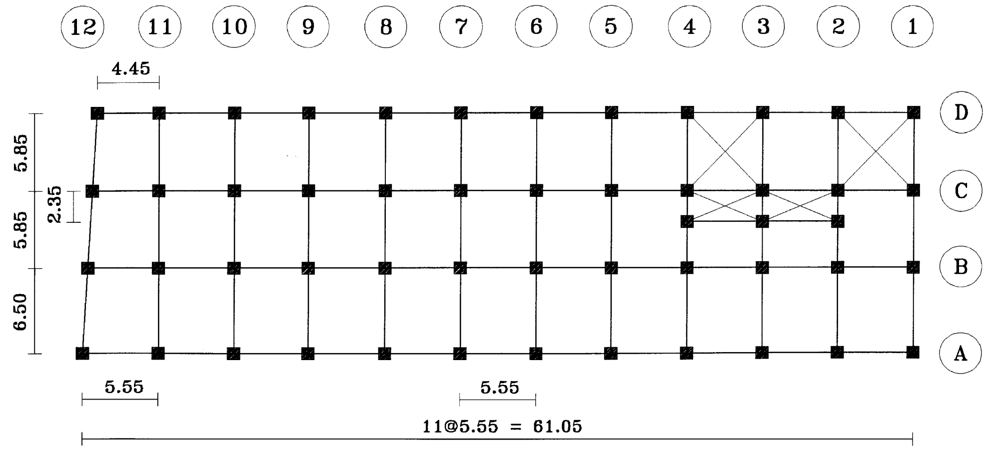

Figure 1. Plan view of the original building (dimensions in meters)

storeys and a total height of 44 m. The structure under these conditions experienced moderate structural damage during the earthquake, due to its flexibility and torsional response.

RETROFITTED BUILDING WITH ADDITIONAL BRACED FRAMES

Because of the poor performance during the 1985 Michoacán earthquake, the building was retrofitted in 1990 by removing the three-storey addition and by adding stiff, 'Macro' Braced Frames (MBF) as depicted in plan in Figure 2 and in elevation in Figure 3. Each MBF has a storey height equivalent to four storeys of the original frames. Two exterior MBF were used in the longitudinal direction and four MBF were used in the transverse direction (Figures 2 and 3). The design of the MBF was done considering that these frames should carry 100 per cent of the seismic forces according to the requirements for seismic design stipulated by Mexico's 1987 Federal District Code (RCDF-87).[14,15] A seismic response modification factor for global ductility of two ($Q = 2$) was assumed for the design of the MBF, according to the provisions of RCDF-87.

Columns of the MBF are of box cross-section made by welding four A-36 steel plates. The MBF exterior columns A*1*, A*5*, A*8*, A*12*, D*1*, D*5*, D*8* and D*12* (Figures 2 and 3) measure 35×35 cm with plate thicknesses of 1·91 cm for the first storey and 0·95 cm for the remaining storeys. Exterior columns B1*, B12*, C1* and C12* measure 30×30 cm with plate thicknesses of 0·95 cm over the entire height of the building. Interior columns B5*, B8*, C5* and C8* were made by connecting two IPR sections (I sections made by welded plates) measuring $h = 30$ cm, $b_f = 15$ cm, $t_f = 1{\cdot}27$ cm and $t_w = 0{\cdot}95$ cm with latticed plates 0·95 cm thick to make a 30×30 cm open box-section.

Beams of the new MBF are IPR sections measuring $h = 40$ cm, $b_f = 25$ cm, $t_f = 1{\cdot}59$ cm and $t_w = 0{\cdot}95$ cm for the transverse frames (1*, 5*, 8* and 12*) and $h = 25$ cm, $b_f = 14{\cdot}6$ cm, $t_f = 1{\cdot}27$ cm and $t_w = 0{\cdot}95$ cm for the longitudinal frames (A* and D*). The diagonal steel braces were made with four welded plates to form a 35×35 cm box section, with thicknesses of 0·95 cm for frames A* and D* and 1·59 cm for frames 1*, 5*, 8* and 12*. Connection details for this retrofit scheme are rather complex and difficult to build, so special inspection in the field was required to construct the retrofit successfully. A view of the retrofitted building is presented in Figure 4.

The existing foundation system did not need to be strengthened as the removal of the upper three storeys compensated for the increased axial forces due to the additional braces, according to the review done by the structural design firm.[14]

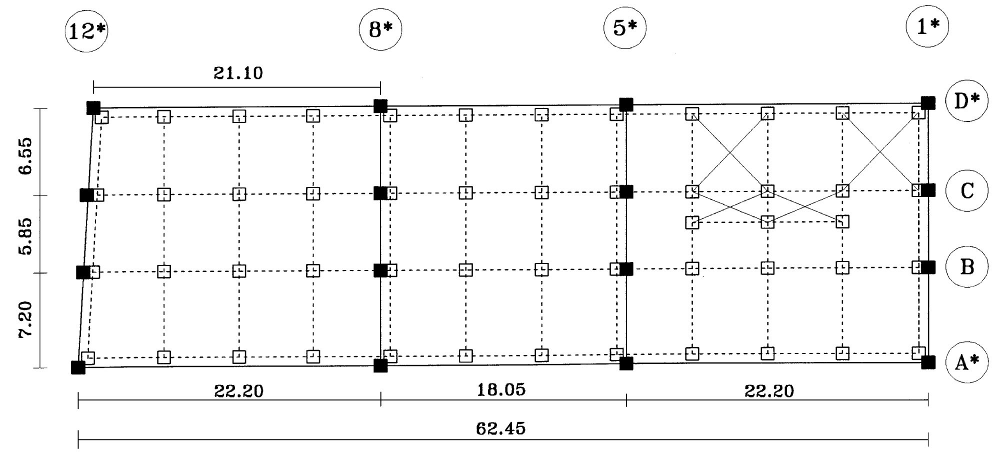

Figure 2. Plan view of the existing MBF retrofit (dimensions in meters)

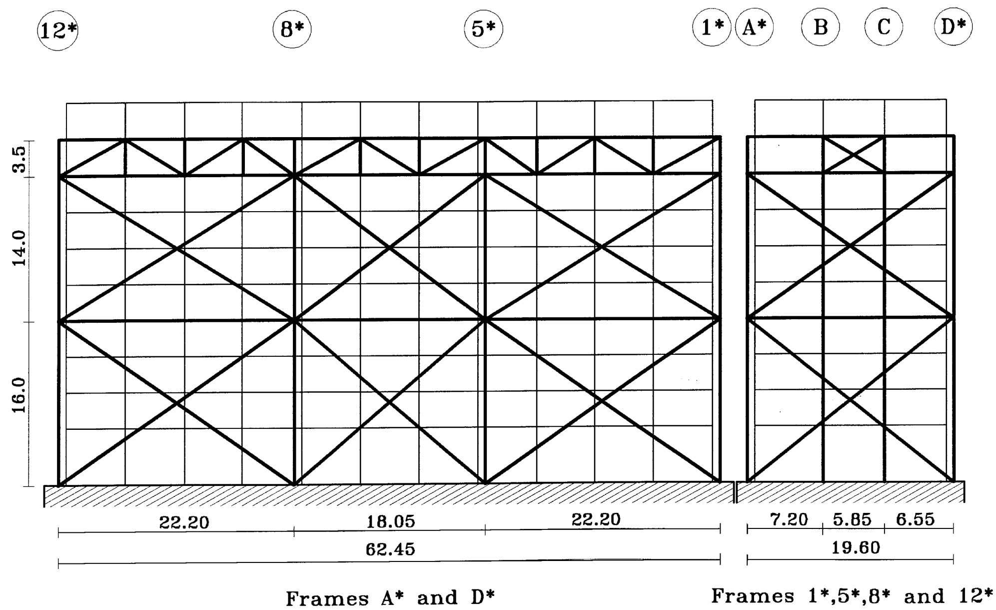

Figure 3. Elevations of the existing MBF retrofit (dimensions in meters)

ALTERNATE RETROFIT WITH ADAS ENERGY DISSIPATION DEVICES

A retrofit scheme using energy dissipation devices could have been a sound technical solution for this building too, because the poor performance of the original building during the 1985 Michoacán earthquake was likely triggered by resonant response. Thus, a theoretical retrofit scheme using ADAS devices was planned for this building to compare against the existing MBF retrofit scheme described in the previous sections. The retrofit with ADAS devices was designed to strengthen the structure in the same areas (frames) covered by the MBF option, as can be seen by comparing Figures 5 and 6 (ADAS retrofit) with Figures 2 and

Figure 4. Northeast view for the subject building

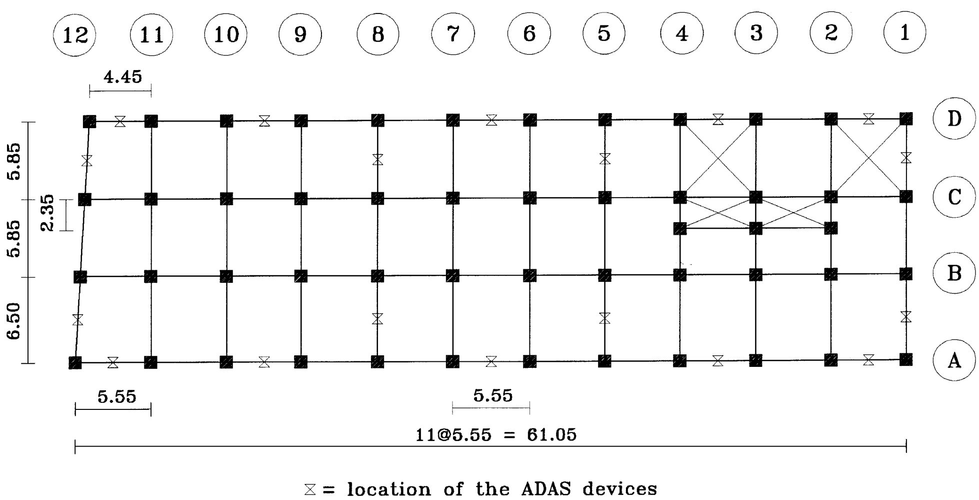

Figure 5. Plan view of the alternate ADAS retrofit (dimensions in meters)

3 (MBF retrofit). The three-storey appendix was also removed as part of this solution. The ADAS devices were assumed to be mounted on chevron steel braces (Figure 6).

Because there are no recommendations for the original design or retrofit of buildings with energy dissipators available in RCDF-93, the design of this retrofit was based on research experience while still observing the general seismic provisions of the RCDF-93 code. The current RCDF-93 code is essentially the RCDF-87 code. It has been shown in experimental research studies[6,7] that ADAS energy dissipation devices increase the global ductility and energy dissipation characteristics of traditional structural systems. Because the retrofit plan with ADAS devices was designed to carry at least 60 per cent of the lateral forces, then, it is reasonable to use the highest response modification factor allowed by the RCDF-93 code for the design of the ADAS devices and the braces, because this type of system has more ductility than traditional lateral load

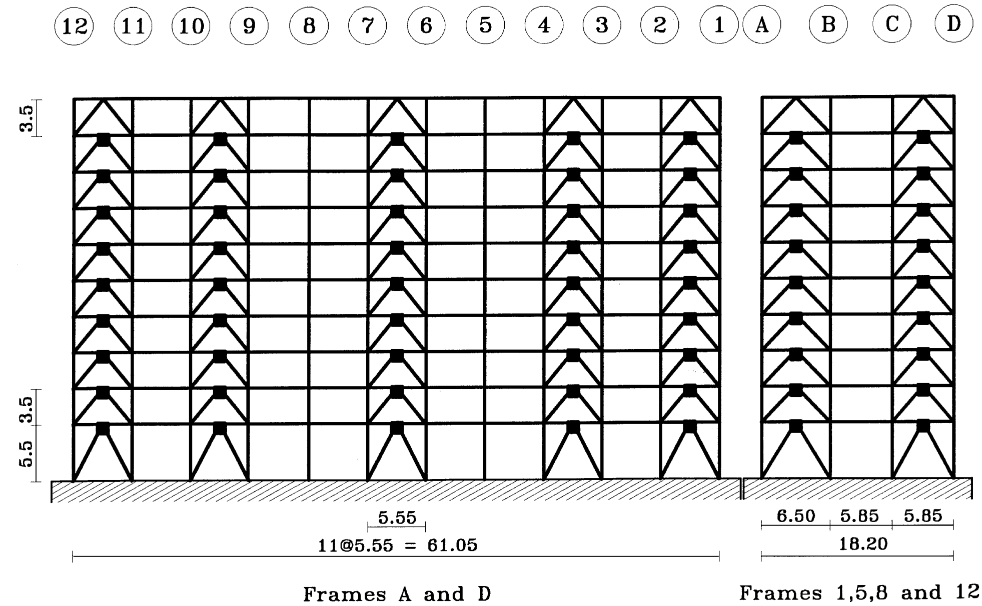

Figure 6. Elevations of the alternate ADAS retrofit (dimensions in meters)

resisting systems and the Q factors in Mexican codes are mostly based on displacement ductility. Therefore, a response modification factor of $Q = 4$, the highest allowed in RCDF-93, was used for the design of the ADAS retrofit.

A 3-D ETABS[16] model of the retrofitted building was used (Figure 7) for design purposes. The design of the retrofit was done by an iterative procedure using the RCDF-93 design spectra for zone III, $Q = 4$ and the SRSS combination. For ease in the iterative design procedure, the ADAS-chevron bracing system in ETABS was idealized by two equivalent diagonals. The axial stiffness of each equivalent diagonal, K_{eq}, was estimated as

$$\frac{1}{K_{eq}} = \frac{1}{K_{diag}} + \frac{2}{K_{ADAS}} \tag{1}$$

where K_{diag} is the axial stiffness of each brace and K_{ADAS} is the shear stiffness of the ADAS device resolved into the angle of the bracing. The mathematical stiffness and strength modelling of the ADAS devices was done according to a procedure recently proposed in the literature.[10,11]

The design procedure used by the authors starts by proposing diagonal braces capable of carrying the axial forces in the system in the absence of ADAS devices. Then, the initial ADAS devices were designed to carry the shear forces transmitted by the chevron braces at the top joint, that is, where the ADAS elements are to be located. Once there is an initial design for the braces and the ADAS devices, the axial stiffness of the equivalent diagonals is modelled according to equation (1), and the building is analysed again. The ADAS devices are redesigned for the shear forces at the top joint and the braces are redesigned to carry the axial forces of the equivalent diagonal using a safety factor of 1·7 to prevent brace buckling. The safety factor is based upon the capacity criteria for axial steel members used in older versions of the AISC design manuals.[14,17] The design of the ADAS devices and the chevron bracing is therefore reduced to an iterative process where elements are proposed until a convergence criterion is met. The global retrofit plan was checked to comply with the storey drift limits established by the RCDF-93 code.

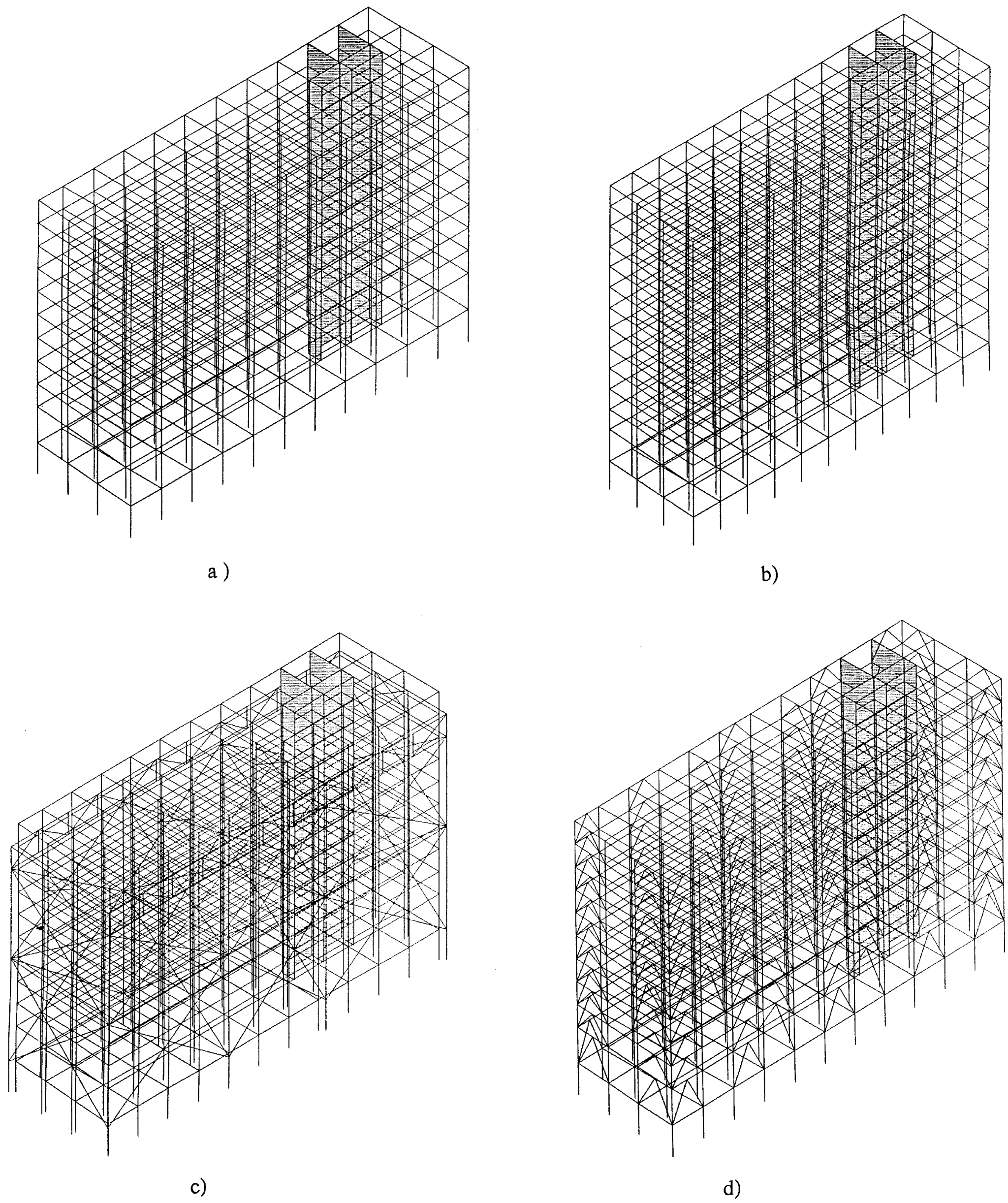

Figure 7. ETABS models of the subject building: (a) model ORIG; (b) model APEN; (c) model MACRO; (d) model ADAS

The ADAS devices were designed as optimum as possible, rounding the number of needed plates to the nearest integer. For example, if the analyses suggested 4·6 plates, the designed ADAS devices consisted of five plates, whereas if the analyses suggested 4·4 plates, the designed ADAS devices consisted of four plates. The reason behind this strategy is that the ADAS devices can develop higher ductilities than those assumed in the design ($Q = \mu = 4$). Then, lower round off design would be associated with yielding of the ADAS device at the same yield displacement but at slightly lower forces than those assumed for design. If a conservative procedure is used for the design of the energy dissipators, then, the original elements of a structure could start

Table I. Designed ADAS devices (dimensions are the depth and thickness of the plates)

Storey	Frames 1, 5, 8 and 12 (N–S)	Frames A and D (E–W)
1	AD2 = 7, $30{\cdot}5 \times 4{\cdot}45$ cm ($12'' \times 1\frac{3}{4}''$) plates	AD3
2	AD1 = 9, $30{\cdot}5 \times 4{\cdot}45$ cm ($12'' \times 1\frac{3}{4}''$) plates	AD3
3	AD2	AD3
4	AD3 = 6, $30{\cdot}5 \times 4{\cdot}45$ cm ($12'' \times 1\frac{3}{4}''$) plates	AD5 = 7, $24{\cdot}1 \times 3{\cdot}81$ cm ($9\frac{1}{2}'' \times 1\frac{1}{2}''$) plates
5	AD3	AD5
6	AD4 = 5, $30{\cdot}5 \times 4{\cdot}45$ cm ($12'' \times 1\frac{3}{4}''$) plates	AD6
7	AD6 = 4, $24{\cdot}1 \times 3{\cdot}81$ cm ($9\frac{1}{2}'' \times 1\frac{1}{2}''$) plates	AD8 = 6, $17{\cdot}8 \times 3{\cdot}18$ cm ($7'' \times 1\frac{1}{4}''$) plates
8	AD6	AD9 = 5, $17{\cdot}8 \times 3{\cdot}18$ cm ($7'' \times 1\frac{1}{4}''$) plates
9	AD7 = 3, $24{\cdot}1 \times 3{\cdot}81$ cm ($9\frac{1}{2}'' \times 1\frac{1}{2}''$) plates	AD10 = 4, $15{\cdot}24 \times 2{\cdot}54$ cm ($6'' \times 1''$) plates

yielding before the dissipators, taking away the benefits of energy dissipation. Table I summarizes the final design of the ADAS elements. A total of 162 ADAS devices is needed for the proposed retrofit scheme. ADAS device AD1 is in fact two devices, one composed of four plates and the other of five plates, because of construction constraints.[17] The chevron bracing design consisted of square cross sections made by 2CPS $305 \times 44{\cdot}64$ for storeys one to three, 2CPS $305 \times 37{\cdot}20$ for storeys four to seven and 2CPS $254 \times 22{\cdot}76$ for storeys eight to ten. In the tenth storey there are no ADAS devices, as the design procedure suggested that the chevron bracing alone was enough because the low drift deformations would not permit commercial sizes of ADAS devices to dissipate energy in this level.

For the design of retrofitted buildings it is also important to check the state of the original elements, reviewing their stress conditions using suitable response modification factors according to the provisions of the design code. For the subject building, a response modification factor as high as $Q = 4$ could have been used for checking stresses of all members of the original moment frames, according to RCDF-93. However, the original steel members of the strengthened and unstrengthened frames were checked using $Q = 3$ using 3-D response spectra analyses of the final retrofit model according to the RCDF-93 code provisions. According to this analysis, the original elements did not need to be strengthened.

ACCELERATION RECORDS

Artificial acceleration records were generated for the Alameda Park site for the 1985 Michoacán Earthquake. The artificial accelerograms were obtained according to a procedure proposed by Ordaz *et al.*[18] to generate acceleration records at any site in Mexico City based upon the transfer function of the site, the strong motion data recorded at more than 100 stations in the last seven years, and the accelerograms of a reference station (usually CU station, hard soil site) for the earthquake of interest. The artificial accelerograms for the E–W and N–S components and their associated response spectra for 2 per cent viscous damping are presented in Figure 8. Peak ground accelerations for the site were $0{\cdot}18g$ in the E–W direction and 0·15g in the N–S direction. The response spectra curves suggest peak dynamic responses for structural systems with associated natural periods in the 1·7–2·3 s period range (Figure 8).

3-D ELASTIC ANALYSES

To assess the initial, elastic dynamic characteristics of the different structural models for the subject building, 3-D frequency analyses of each model were done in ETABS. The ETABS models are depicted in Figure 7. In these analyses, the following assumptions were made: (a) Young's modulus for structural steel was taken as $E_s = 2{\cdot}0529 \times 10^5$ MPa ($2{\cdot}0139 \times 10^6$ kg/cm^2); (b) because of the presence of several welded plates at the top and bottom of the beam-column connections (with an overall thickness of around 5 cm at each end and

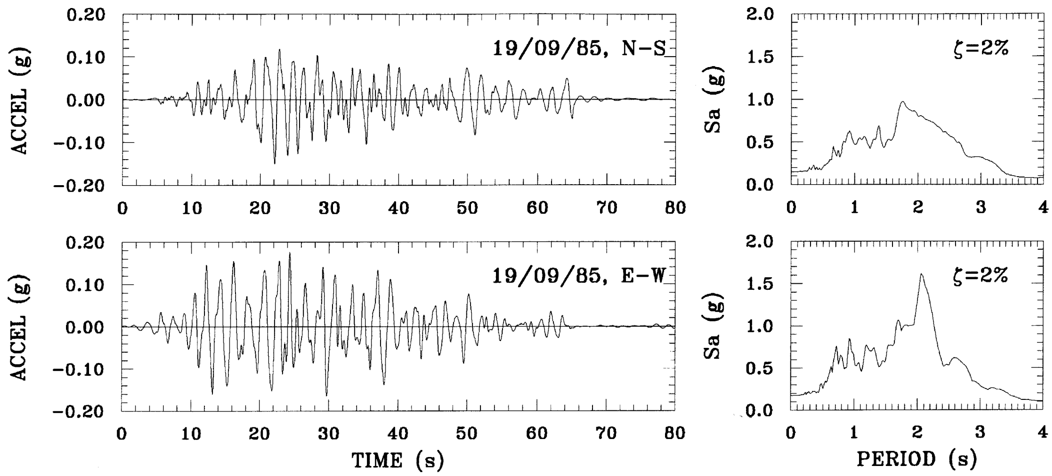

Figure 8. Simulated acceleration records for Alameda Park site for the 1985 Michoacán Earthquake

Table II. Dynamic characteristics of the 3-D models under study

Model	W	Mode	Period (s)	Modal mass (%) N–S	Modal mass (%) E–W	Modal mass (%) Rotation
Original	8784 ton	1. Coupled	1·96	26·19	37·82	13·73
		2. Coupled	1·83	25·00	42·46	11·07
		3. Coupled	1·17	26·95	0·03	47·89
Appendix	11 198 ton	1. Coupled	2·64	37·97	17·09	19·55
		2· Coupled	2·43	11·26	62·36	4·78
		3. Coupled	1·61	25·67	0·02	47·07
MBF	9015 ton	1. N–S	0·90	78·12	0·44	0·65
		2. E–W	0·81	0·42	89·21	0·02
		3. Torsion	0·54	0·92	0·05	72·00
ADAS	8907 ton	1. N–S	1·19	55·16	13·49	7·19
		2. E–W	1·24	10·85	64·44	2·03
		3. Torsion	0·86	10·11	0·05	63·01

lengths from 45 to 70 cm) that make the joints stiffer than usual, the rigid-end zones were extended 50 cm from the beam face to account for this condition; (c) the masonry walls that form the elevator core were included in the modelling and (d) soil–structure interaction effects were modelled according to the recommendations of RCDF-93, considering that foundation piles are supported on the hard-rock basin of Mexico City.

Results from the frequency analyses are summarized in Table II, where the total weight (dead load plus reduced live load) estimated for each model is also provided. Many observations can be drawn from this information. Regarding the weight of each structure, it can be observed that the three-storey addition ('appendix' model) increased the weight of the structure by 27·5 per cent with respect to the original structure. However, the weight of the retrofit models was only slightly increased with respect to the original structure (2·6 per cent for the MBF model and 1·4 per cent for the ADAS retrofit).

The mode shapes for the original and 'appendix' models are highly coupled, with important torsional components. The coupling in the mode shapes is due to the presence of the eccentric elevators core walls and the distribution of the original elements. The coupling of the 'appendix' model is lower than for the original model. The natural periods for the original structure are in the 1·83–1·96 s range, where resonant responses can be expected for the Alameda Park site (Figure 8). It can be assumed that if the original structure had been present during the 1985 earthquake, it may have suffered severe structural damage or collapsed because of resonant response with the site, as is shown in the following nonlinear dynamic analyses, in addition to the torsional response associated with the coupled modes. The natural periods for the appendix model are in the 2·43–2·64 s period range, which are on the descendant branch of the response spectra for the site (Figure 8). Thus, perhaps the construction of the additional three storeys may have helped the building to survive the 1985 earthquake, putting the structure in a reduced range of response with respect to the original structure (Table II, Figure 8). Nevertheless, because of its flexibility and the torsional coupling, the structure experienced high deformations that caused the damage observed after the 1985 earthquake, as is shown in the following nonlinear dynamic analyses. Therefore, a retrofit of the structure that survived the 1985 earthquake was needed to improve its stiffness, strength and dynamic characteristics.

The existing retrofit with MBF considerably increased the lateral stiffness of the structure in both directions and substantially improved its dynamic characteristics (Table II). The natural periods (0·90 s for the N–S direction and 0·81 s for the E–W direction) are well below the region for resonant responses with the site (Figure 8). In addition, the translational mode shapes are almost pure as the stiffening almost eliminated the torsional coupling (Table II). The alternate retrofit with ADAS devices is apparently less effective than the MBF retrofit, from the frequency analysis viewpoint, but adequate to take the structure away from resonant responses. The natural periods, 1·19 s for the N–S direction and 1·24 s for the E–W direction are reasonably below the region for resonant responses with the site (Figure 8). The modes are cleaner than for the original and appendix structures, but they are still coupled.

LIMIT ANALYSES

Limit analyses of different idealizations of the subject building (original, appendix, MBF and ADAS) were carried out to estimate their ultimate capacity when subjected to lateral loading. A lateral force distribution according to the fundamental mode shape of the direction under study was used in the analyses. Because all beams are compact sections, they were assumed to develop their full plastic capacity. For the columns, the ultimate stress was computed to take into account their slenderness and their brace and support conditions. Thus, all the columns for the original and appendix models were considered unbraced. The columns that are in the braced frames for the MBF and ADAS models were considered braced (sidesway inhibited) whereas the columns of the unbraced frames were considered as leaned columns. Also, it was considered that the floor system and the deep plates restrained the deformation of the columns when computing the restraint factors G. For the ADAS and MBF models, the studied mechanisms considered the failure of the braces working in compression and neglecting the contribution of the braces working in tension, a conservative approach to the ultimate capacity of the frames that is justified because lateral loads alternate during an earthquake event. For the ADAS models, the shear failure of the ADAS devices was included in the formulation.

Several failure mechanisms were studied for each idealization, including the weak beam–strong column mechanism and soft-storey failure mechanisms. The resulting failure mechanisms are summarized in Table III. For the original structure, a weak beam–strong column (wb–sc) mechanism governed for the N–S direction whereas a combined failure mechanism with hinges at the base of the columns and at the top of the eighth-storey columns and all beams from storeys one to seven ruled for the E–W direction. For the appendix model, the failure mechanism for both the E–W and N–S directions was the same combined mechanism as described above. Similar combined failure mechanisms controlled the ultimate lateral load capacities of the ADAS retrofit. For the MBF, the dominant failure mechanisms are associated with combined failure mechanisms for the first macro frame storey, that is, the first four storeys of the original structure. It can be

Table III. Ultimate base shear from limit analyses vs. design base shears

Model	Direction	Failure mechanism	Ult. Shear	RCDF-42	RCDF-66	RCDF-76	RCDF-87
Original	N–S	wb–sc	0·108W	0·025W	0·060W	0·060W	0·100W
	E–W	Combined	0·092W	0·025W	0·060W	0·060W	0·100W
Appendix	N–S	Combined	0·074W	—	0·060W	0·060W	0·100W
	E–W	Combined	0·063W	—	0·060W	0·060W	0·100W
MBF	N–S	Macro 1st storey	0·352W	—	—	—	0·200W
	E–W	Macro 1st storey	0·299W	—	—	—	0·200W
ADAS	N–S	Combined	0·191W	—	—	—	0·100W
	E–W	Combined	0·275W	—	—	—	0·100W

concluded that the failure mechanisms are not the best, thus, attending to the limit analyses, neither retrofit scheme helps substantially improve the nature of the collapse mechanism of the structure.

Ultimate base shear capacities associated with the failure mechanisms described above are summarized in Table III, where they are compared against the design base shear for different versions of Mexico's Federal District Design Code (RCDF), taking into account the response modification factor 'Q' allowed by the code according to the structural system for each case. In Table III, 'W' stands for the total weight of each structure, according to the values given in Table II. It can be observed that the original structure satisfied the strength requirements established by the ruling code at the time of construction (RCDF-42) and still satisfies the 1976 code ($Q = 4$), but cannot entirely satisfy the 1987 code requirements ($Q = 4$) that are the same as the current 1993 code. The appendix structure satisfies the 1976 code requirements ($Q = 4$), but cannot satisfy the 1987 and 1993 code requirements ($Q = 4$). The lateral strength of the appendix structure is reduced when compared with the original structure, thus, looking at this aspect of response, the three-storey appendix had a negative impact.

The building in its current condition (MBF) satisfies the strength requirements of the 1987 and 1993 codes by a large margin, taking $Q = 2$ as allowed by the code. The additional macro braced frames carry 71 per cent of the shear forces in the E–W direction and 70 per cent in the N–S direction.[19] The proposed upgrade with the ADAS devices also satisfies the 1987 and 1993 codes as a higher response modification factor would be allowed for this solution in both directions ($Q = 4$). The retrofitted frames with ADAS devices carry 84 per cent of the shear force in the E–W direction, whereas in the N–S direction they carry 61 per cent of the shear force.[19]

NON-LINEAR DYNAMIC ANALYSES

To help rationalize the structural damage that the appendix building experienced during the 1985 Michoacán earthquake, and the dynamic response that could be expected for the original, MBF and ADAS models, representative frames for the building in the N–S direction were analysed using the artificial N–S accelerogram presented in Figure 8 (simulated for this earthquake) and the DRAIN-2DX computer program.[20] The 2-D non-linear dynamic analyses were conducted in lieu of rigorous 3-D non-linear dynamic analyses or 2-D analyses that would take into account the interaction between frames because of software and computer hardware limitations, which also limited the study to the analyses of relatively short frames. Nevertheless, these analyses can give valuable insight to the dynamics of the subject structures, especially when the critical direction seems to be the N–S direction. The critical frames for the 2-D analysis, according to the 3-D ETABS and the limit analyses, were frames 8 and 8* (Figures 2, 3, 5 and 6).

Four models were studied. Model ORIG is the original structure. Model APEN represents the 'appendix' structure as it was during the 1985 earthquake. Model MACRO is the current MBF retrofitted structure.

Finally, model ADAS is the retrofitted structure with ADAS devices. The initial fundamental periods for the DRAIN-2DX models were those determined for the structure by the ETABS models in the direction of interest (Table II) by computing equivalent dynamic masses for the frames according to the 3-D distribution of forces associated with the corresponding dominant mode shape. Soil-structure interaction was modelled by elastic spring elements. Initial stresses due to dead and reduced live loads carried by the frame were included. An equivalent stiffness-proportional viscous damping equal to 2 per cent was included in the analyses. Nominal plastic strengths of the steel beam and column members were considered as was done in the limit analyses. Rigid end zones were included. A 1 per cent post-yield stiffness was assumed for all elements. For the models with diagonal bracing (MACRO and ADAS), steel braces were modelled as non-linear axial elements with elastic buckling in compression and elastic-perfectly–plastic behaviour in tension. Nominal axial capacities were computed according to the steel provisions of the Mexican code, which gives values similar to those computed with the AISC-LRFD code.[21] The ADAS elements were modelled by equivalent beam–column elements whose heights equal the ADAS' height. These equivalent elements were connected to the chevron braces and beams, letting the connecting nodes rotate freely, as the end-fixity commonly assumed for the ADAS devices is not achieved, even in lab tests.[6,10,11] The strength modelling of the ADAS elements was done so that they could develop their ultimate shear capacity, according to a procedure outlined elsewhere.[10,11]

Dynamic results processed from the analyses were peak dynamic storey drift angles ($\delta = \Delta_i/H_i$), maximum dynamic storey shear indexes (V/W_{T}), storey hysteresis curves (V/W_{T} vs. δ) and yielding mapping for time-steps associated with peak dynamic responses. W_{T} is the estimated weight of the original building. Peak dynamic storey drift angles and maximum dynamic storey shear indexes are depicted in Figure 9 for the referenced models. In Figure 9, RDF-a represents the limiting drift angle of the current Federal District Code (RCDF-93) for a structure that could have non-structural elements not properly separated from the structural system ($\delta = 0{\cdot}006$), whereas RDF-b is the maximum drift angle allowed by the code when non-structural elements are properly separated from the structural system ($\delta = 0{\cdot}012$). In Figure 9 and successive related figures in the text, $W_{\mathrm{T}} = 8784$ ton, the estimated weight of the original building (model ORIG).

The analyses for model ORIG suggest that the original structure could have collapsed during the 1985 earthquake, as significant deformation, strength and ductility demands are detected for the frame under study (Figures. 9 and 10). The peak dynamic storey drifts are higher than the code limit RDF-a from storeys four to ten and much higher than code limit RDF-b for the first three storeys. The original frame had clear weaknesses in the first three storeys, where significant drift angles and ductility demands were detected (Figures 9 and 10). Ductility capacities of at least three to five are needed in these storeys to achieve dynamic stability. The detailing of the original columns and beams and the existing riveted connections could not achieve these levels of deformation without substantial damage. In fact, generalized hinging of the first storey columns was detected at the time-step associated with the peak dynamic drift, suggesting a potential storey column collapse mechanism. The peak dynamic base shear for this frame ($V = 0{\cdot}011 W_{\mathrm{T}}$) is practically the same as that predicted by the limit analyses when assuming a first storey column mechanism ($V = 0{\cdot}0109\, W_{\mathrm{T}}$) and slightly higher than that associated with the critical collapse mechanism identified in the limit analyses ($V = 0{\cdot}0095\, W_{\mathrm{T}}$), thus confirming the usefulness of limit analyses in accurately predicting the lateral load capacity of typical unbraced frames despite the crude assumption of the lateral load pattern for the limit analyses.

The analyses for model APEN suggest that the structure could have experienced substantial damage due to large deformations if subjected to the subject ground motion associated with the 1985 earthquake, particularly in storeys one to three and eight to ten (Figures 9 and 11). However, the ductility, strength and deformation demands are smaller than for model ORIG in the lower five storeys (Figure 9). Drift angles were high. The peak storey ductility demand was about 3·6 for the first storey. Dynamic yielding at times of peak dynamic deformations and strength demands correlated reasonably well with the combined collapse mechanism identified in the limit analyses, although hinging was detected for the beams of storeys one to ten

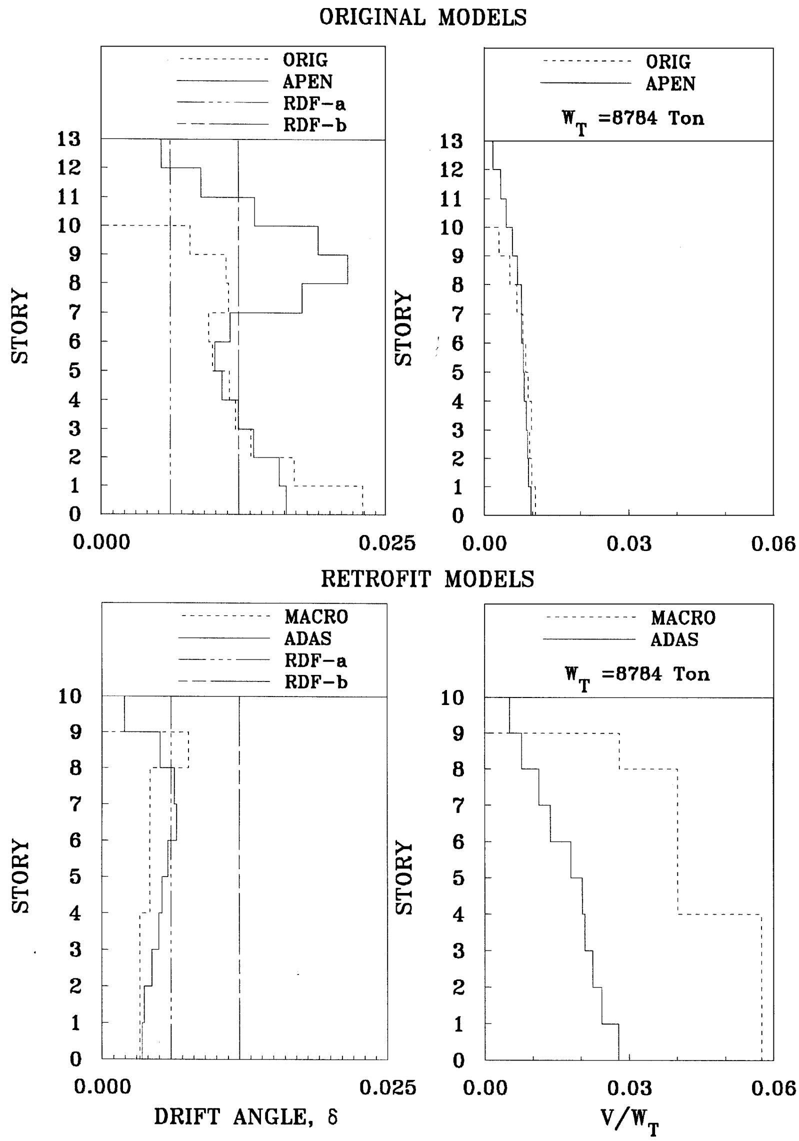

Figure 9. Maximum response envelopes

in the dynamic analysis instead of storeys one to seven in the limit analysis. Therefore, the peak base shear for the dynamic analyses ($V = 0{\cdot}0095W_T$) was higher than that predicted by the limit analysis for this frame ($V = 0{\cdot}0065W_T$), as the ground record excited other failure modes that provided additional strength. Overall, the dynamic analyses suggest that adding three storeys could have actually helped the structure survive the 1985 Michoacán earthquake despite the observed structural damage, as the dynamic response for model APEN is better than for model ORIG.

The dynamic analyses for the retrofit models MACRO and ADAS confirm the benefit of the retrofit approaches in improving the seismic behaviour of the subject building. Peak dynamic drifts angles are within reasonable limits according to the RCDF-93 code (Figure 9). Peak dynamic drifts are slightly higher for the ADAS model than for the MACRO model. However, the peak dynamic storey shears for the ADAS retrofit

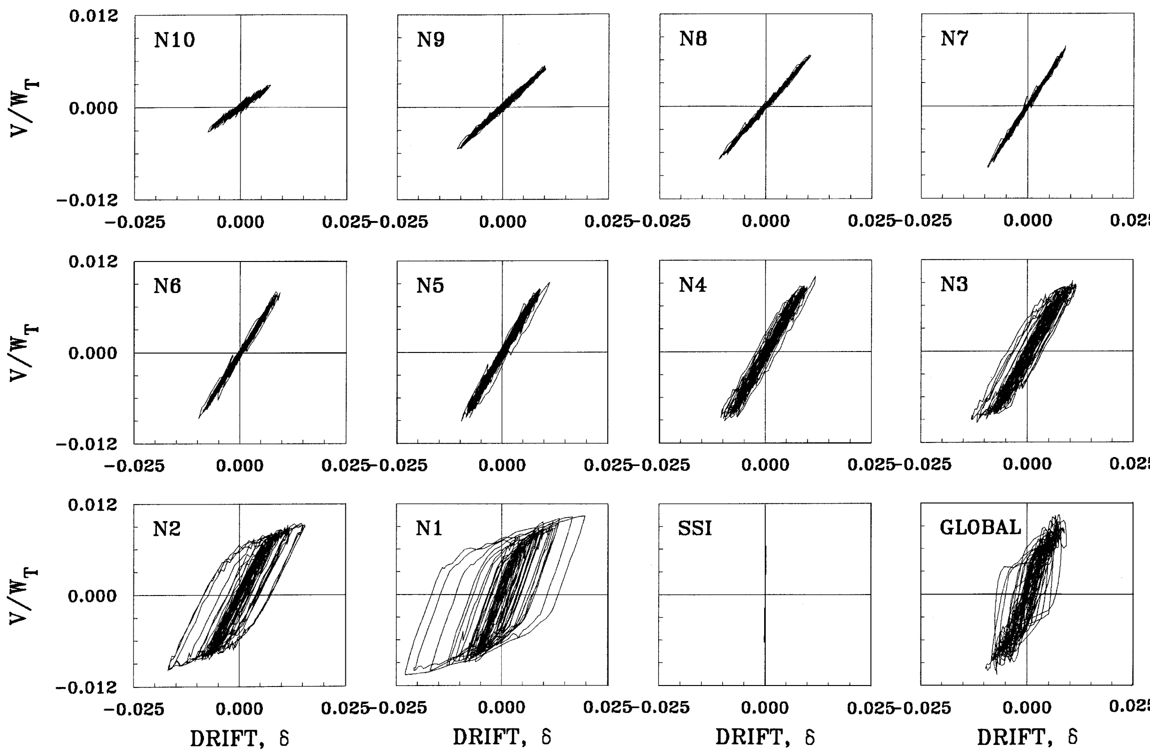

Figure 10. Hysteresis curves for model ORIG

are less than half of those for the MACRO model (Figure 9). As the pseudo-acceleration response spectrum for 2 per cent damping for the N–S record (Figure 8) yields similar values for the elastic natural period of both models, it is clear that the ADAS retrofit yields smaller storey shear forces because of the energy dissipation provided by the ADAS devices, which can be also understood as a higher equivalent internal damping. Also, the axial forces transmitted to the foundation are less than half of those transmitted by the MACRO retrofit, so the ADAS retrofit is more advantageous than traditional stiffening with bracing in this regard. If foundation retrofitting had been required for this building, the ADAS solution may have been a better solution as foundation strengthening is usually very expensive to implement in buildings of this size, and the ADAS solution may have avoided the need for foundation strengthening.

The improved dynamic performance of the retrofit with ADAS devices over the existing MBF option can be confirmed with the hysteresis curves depicted in Figures 12–14. It can be observed in Figure 12 that the energy dissipation for model MACRO is limited and is due primarily by column yielding and dynamic brace buckling. In fact, the bracing for levels five to eight (N5–N8) buckle dynamically in compression over about 37 s out of 66 s of non-linear response, with the risk of being permanently damaged. This brace buckling is the reason why the hysteresis curves for levels N5–N8 are irregular. The dynamic buckling of the braces is made worse by the fact that they are very slender ($kl/r > 100$). Therefore, this study suggests the use of braces with short to intermediate slenderness ratios in seismic regions (that is, $kl/r < 100$), to diminish the potential threat of brace buckling, which has also been observed in the 1994 Northridge earthquake. The studied MACRO frame still has a reasonable reserve capacity, as an ultimate base shear capacity of $0{\cdot}063W_T$ was conservatively computed for this frame using limit analysis, where the strength of the braces in tension was neglected.

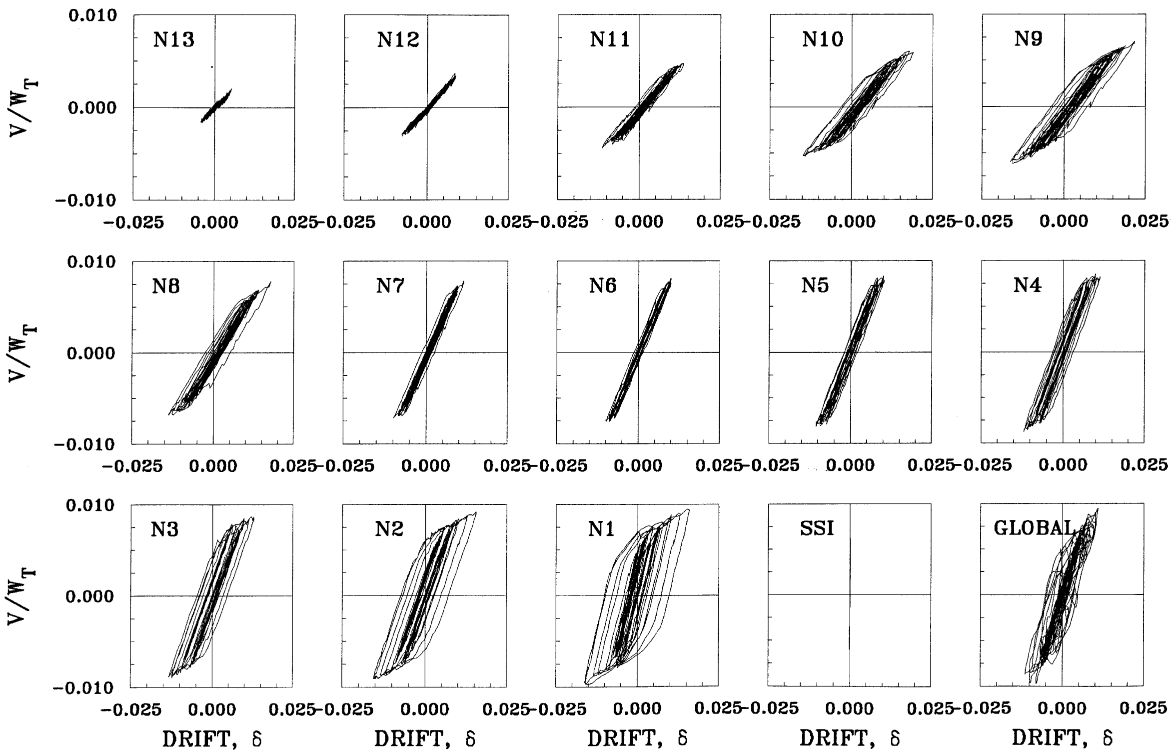

Figure 11. Hysteresis curves for model APEN

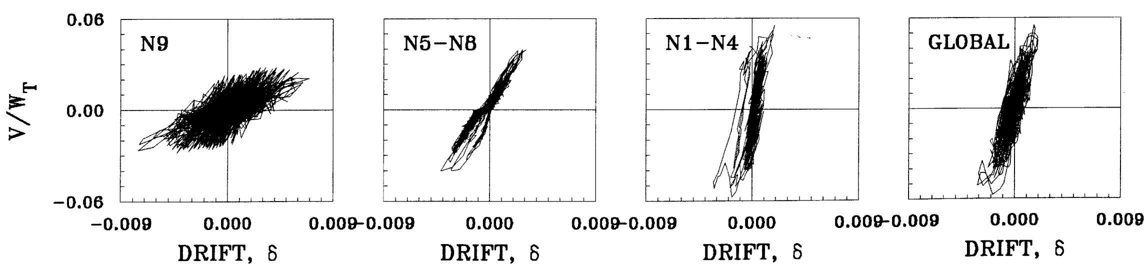

Figure 12. Hysteresis curves for model MACRO

In contrast to what was observed for the MBF retrofit, energy dissipation in the ADAS retrofit is high and stable, as depicted in Fig. 13. Storey ductility demands range from 1·8 to 3·8, in the range assumed for design (Figure 13). The ductility and energy dissipation is mostly provided by the ADAS devices, as can be observed from the hysteresis curves of the ADAS devices depicted in Figure 14, which are normalized with respect to the yield shear capacity of the ADAS devices of the first storey. In Figure 14, Δ is the ADAS' drift computed as the difference between the displacements for the top and bottom nodes of the ADAS element over the height of the ADAS device. It can be observed that the ADAS element dissipate substantial energy in a stable way, with local ductility demands that range from 1·8 (N2) to 5·3 (N7). ADAS devices can develop ductility demands as high as ten without experiencing strength degradation.[6,22] Also, for the ADAS retrofit, the ADAS-chevron system carries from 52·5 to 92·0 per cent of the total storey shear, thus, alleviating the

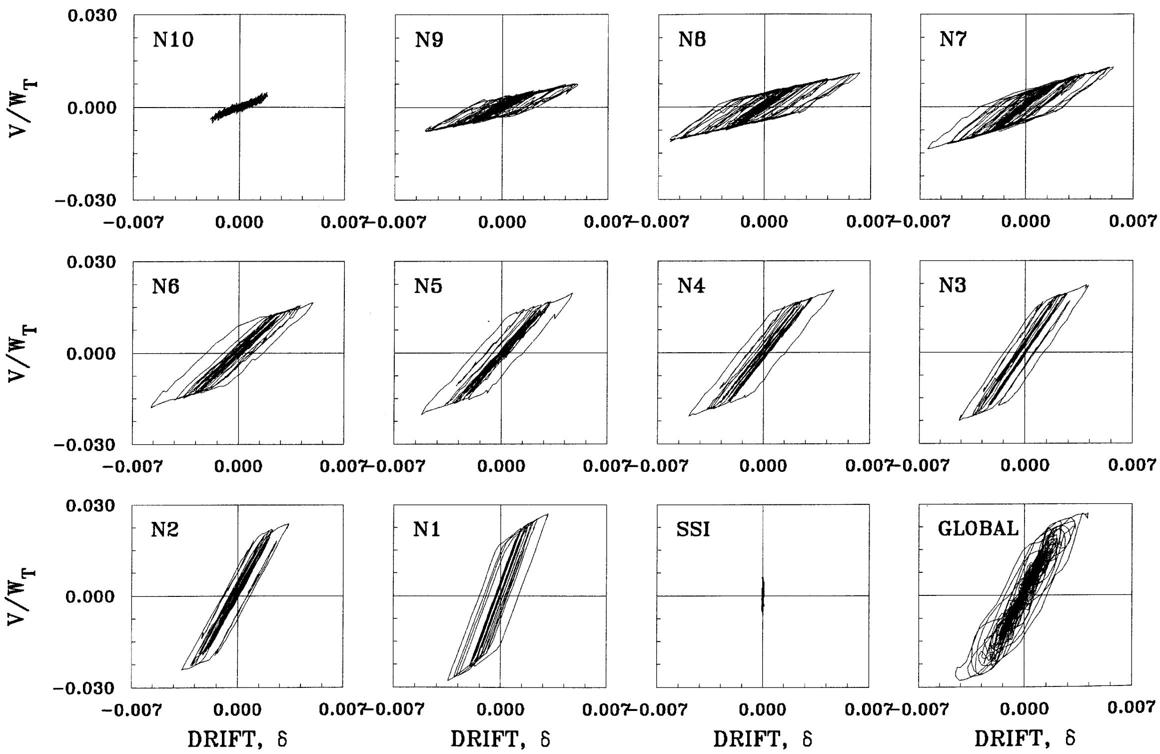

Figure 13. Hysteresis curves for model ADAS

demand on the original elements. In fact, at the time of peak dynamic response, the only elements that yield under the ADAS retrofit model are the ADAS elements and the central beams for storeys five and six. In addition, the ADAS frame still has reserve capacity, as an ultimate base shear capacity of $0{\cdot}036W_T$ was conservatively computed for this frame using limit analysis, where the strength of the braces in tension was neglected. Overall, the retrofit with ADAS energy dissipation devices has much better dynamic performance characteristics than the constructed MBF retrofit, as has been shown in the present study.

INITIAL COST OF RETROFIT ANALYSIS

An important issue in the decision process to start a retrofit plan is the cost analysis. Unfortunately for structural engineering practice in Mexico, life-cycle cost analyses are seldom weighted in this process as the building owners always want to have their facility readily available as soon as possible at the cheapest initial cost. Thus, the initial cost of retrofit is an important issue to weight when analyzing different retrofit options. For this building, the initial cost of retrofit would include the cost of erecting the additional structural elements for each case, the demolition costs of the upper three storeys, including the relocation of the elevators' machinery, and the cost of improvements to the facilities and aesthetics of the building. The foundation did not need to be strengthened. Nevertheless, the present initial cost study will only include the cost associated with the retrofit of the superstructure because of the following reasons: (1) the demolition costs would be the same for both retrofit alternatives, and no reliable information was obtained on this regard, and (2) the cost of improvements to the facilities and aesthetics are hard to estimate, as these are highly owner-architect dependent.

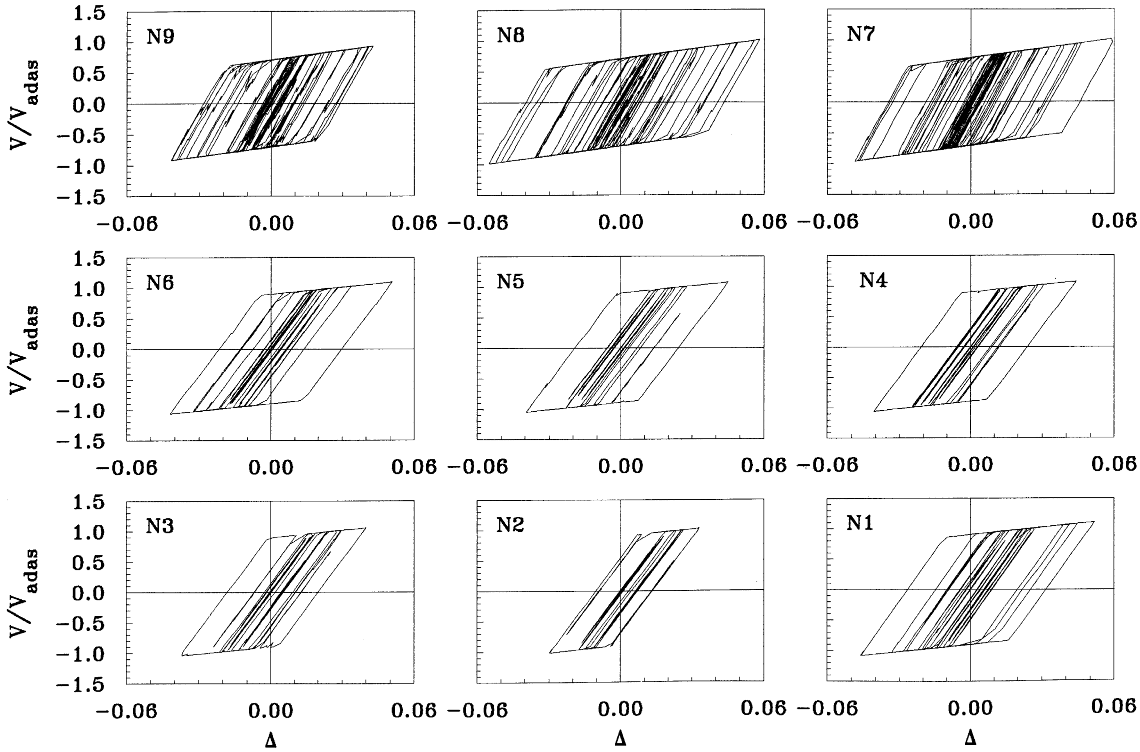

Figure 14. Hysteresis curves for the ADAS elements for bay A-B, ADAS model.

The superstructure cost of the existing MBF retrofit involves 285 t (2795·9 kN) of structural steel, including a 15 per cent waste because of the complexity of the connections. Thus, considering the cost of mounted structural steel that includes labor force, mounting, local and trade taxes, non-corrosive paint and welding, the initial cost of retrofit for the MBF due to the superstructure is 1·00 in relative cost terms. For the ADAS retrofit, a total of 145 t (1422·5 kN) of structural steel is needed, including a 10 per cent waste because of the connections. Thus, the initial relative cost due to structural steel was 0.51. In addition, 162 ADAS devices are needed. The company that trades the ADAS in Mexico City estimated a relative cost of 1.40 for the ADAS, including installation costs, transportation costs, trade taxes and copyrights. Thus, the total initial cost of retrofit for the ADAS option due to the superstructure is 1·91 times the initial cost for the superstructure of the existing MBF retrofit. In this regard, there is no doubt that the MBF retrofit looks more attractive to the building owner, especially because the foundation did not need to be strengthened. The ADAS retrofit may have been the least expensive option if significant foundation retrofitting had required, as foundation strengthening is very expensive.

SUMMARY AND CONCLUSIONS

A comparative study of an existing retrofit for a mid-rise steel building in downtown Mexico City using additional stiff steel braced-frames against an alternate retrofit using ADAS passive energy dissipation devices was presented. The building was damaged during the 1985 Michoacán Earthquake and was later retrofitted according to the seismic provisions of Mexico's 1987 Federal District Code. Different sets of analyses were carried out to compare both alternatives: (a) three-dimensional elastic analyses; (b) limit

analyses; (c) nonlinear dynamic analyses for postulated site ground motions for a $M_s = 8.1$ earthquake; and (d) initial cost of retrofit analysis.

The comparative studies suggest that a retrofit using ADAS devices would have a superior dynamic performance than the existing retrofit using steel braces (MBF retrofit) for postulated ground motions associated with a $M_s = 8.1$ earthquake. Energy dissipation is effective, leading the structure to reasonable levels of deformation and alleviating the stresses on the original structural elements. The non-linear response is almost exclusively concentrated in the ADAS devices, which are subjected to deformation and strength demands that they can easily achieve. The shear and axial forces transmitted to the foundation are considerably smaller than those for the existing retrofit. The ADAS retrofit also has enough reserve capacity to withstand more severe ground motions than those assumed in the analysis.

Although the existing MBF retrofit provides more strength and stiffens the structure more, its dynamic response is not as good as the ADAS retrofit, as there is potential brace buckling because of slenderness. In fact, this study shows the danger of designing slender braces in earthquake regions, that unfortunately is a common practice, as many structural engineers design the braces as 'tension-only members'. Nevertheless, the existing MBF is a good solution as it leads the structure to reasonable levels of dynamic deformation, eliminates torsional responses and has considerable strength reserve capacity, especially if one considers the additional strength capacity because of three-dimensional response, that could not be evaluated in this study. In addition, the initial cost of retrofit for the existing MBF option is less expensive than the ADAS retrofit.

ACKNOWLEDGEMENTS

Financial support of Secretaría General de Obras del Departamento del Distrito Federal, Mexico City, Mexico, is acknowledged. Appreciation is extended to Eduardo Pérez-Rocha for generating the acceleration records used in this study, to Prof. Oscar De Buen for sharing relevant information on the original structure and the existing retrofit for the subject building and for his critical review of the original manuscript, to Distribuidora de Implementos Sismoresistentes (DIS) for providing cost information on the ADAS devices, to Prof. Enrique Del Valle for his critical review of the original manuscript and the anonymous reviewers of the journal for their valuable observations and suggestions that helped substantially improve this paper.

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